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Reduction of pounding effects in elevated bridges during earthquakes

Robert Jankowski,* Krzysztof Wilde and Yozo Fujino

Department of Civil Engineering, University of Tokyo, Hongo 7-3-1, Bunkyo-ku, Tokyo 113, Japan

SUMMARY

Pounding of adjacent superstructure segments in elevated bridges during severe earthquakes can result in significant structural damage. The aim of this paper is to analyse several methods of reduction of the negative effects of collisions induced by the seismic wave propagation effect. The analysis is conducted on a detailed three-dimensional structural component model of an isolated highway bridge. The results show that the influence of pounding on the structural response is significant in the longitudinal direction of the bridge and significantly depends on the gap size between superstructure segments. The smallest response can be obtained for very small gap sizes and for gap sizes large enough to prevent pounding. Further analysis indicates that the bridge behaviour can be effectively improved by placing hard rubber bumpers between segments and by stiff linking the segments one with another. The experimental results show that, for the practical application of such connectors, shock transmission units can be used. Copyright © 2000 John Wiley & Sons, Ltd.

KEY WORDS: pounding; elevated bridge; earthquake excitation; travelling seismic wave; rubber bumper; crushable device; shock transmission unit

INTRODUCTION

During recent severe earthquakes, pounding of adjacent structural frames was observed in several elevated bridges. In the Loma Prieta earthquake of 1989, collisions between the lower roadway and piers supporting an upper-level deck of the Southern viaduct section at the China Basin in California resulted in substantial damage [1]. In this case, the difference in the natural frequencies of adjacent frames of the bridge was the cause of pounding. Collisions between neighbouring superstructure segments, supported on identical piers, were observed in a highway bridge near Los Angeles which was instrumented with a set of accelerometers [2]. Data records collected during the earthquake showed spikes of magnitude even 10 times higher than the maximum ground motion acceleration. These spikes confirmed the presence of pounding which was induced

*Correspondence to: Robert Jankowski, Department of Structural Mechanics, Faculty of Civil Engineering, Technical University of Gdańsk, ul. Narutowicza 11/12, 80-952 Gdańsk, Poland

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by the seismic wave propagation effect [2]. The reports of damage to highway bridges during the Kobe earthquake of 1995 indicate that pounding, which occurred after destruction of bearing supports, was the cause for local damage and one of the reasons for falling down of superstructures [3].

The earthquake-induced pounding in bridges has been recently studied intensively. Ma and Pantelides analysed collisions between a superstructure and abutment in a short single span bridge using a single-degree-of-freedom structural model [4]. Kawashima and Yabe studied different unseating prevention and energy dissipation devices to suppress the bridge response in order to minimize pounding with the abutment [5]. Jankowski *et al.* used a simplified lumped mass model of the bridge to analyse the influence of pounding between superstructure segments [6]. In that analysis, every segment was discretized as a single-degree-of-freedom system, and the contribution of dynamics of piers to the total structural response was neglected. The results of the study show that pounding may significantly increase the reaction forces at the piers bases and induce large impact forces in the superstructure.

The use of seismic isolation of bridges, which became a common technique to reduce the seismic force, elongates the natural period of the structure, resulting in large displacements and therefore increasing the possibility of pounding. The earthquake design codes [7] specify that the gap size between bridge segments should be large enough to avoid collisions. On the other hand, however, enlarging the expansion joints between girders is expensive and an undesirable solution.

The purpose of this paper is to analyse several methods of reduction of the negative effects of pounding between superstructure segments of an isolated elevated bridge induced by the seismic wave propagation effect. The analysis is conducted on a detailed three-dimensional structural component model of the structure in which deck and piers have been discretized as multi-degree-of-freedom beam-column elements, and bearings are simulated as spring-dashpots. First, the bridge properties, such as the gap size between segments and properties of bearings, are optimized. Then, the effects of additional devices, such as dampers, stiffeners, rubber bumpers and crushable devices, placed between segments are investigated. Finally, the possible application of shock transmission units (STUs) is considered, based on the results of the conducted experiment.

3D MODEL OF ISOLATED ELEVATED BRIDGE

Description of the analysed structure

A base-isolated elevated highway bridge specified according to the 'Manual for Menshin Design of Highway Bridges' [8] is used to study the influence of pounding on the structural behaviour. The longitudinal and transverse cross-sections of the bridge are presented in Figure 1 and the properties of the structural members are summarized in Table I. The deck of the bridge consists of three-span-continuous, prestressed concrete segments with a mass of 2×10^4 kg/m. The length and width of a single superstructure segment are 120 and 14 m, respectively. A substructure consists of identical reinforced concrete piers of height 11.5 m. Two high-damping rubber bearings (HDRBs) support the superstructure at every pier. The cross-sectional area and the thickness of rubber layers in a single bearing are 0.7921 m^2 and 0.082 m, respectively.

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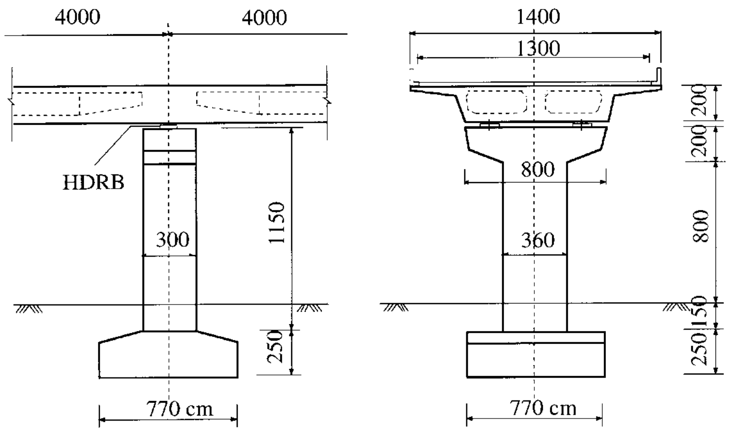

Figure 1. Longitudinal and transverse cross-section of the bridge.

Table I. Properties of bridge structural members.

Structural element	Mass density (kg/m^3)	Mean cross sectional area (m^2)	Modal damping ratio
Superstructure	1176	17	0.03
HDRB	—	0.7921	0.14
Top of pier	3788	15.408	0.05
Pier	3788	10.8	0.05

Modelling of structural members of the bridge

In order to carry out the analysis of the bridge response, superstructure segments and piers have been discretized as elastic 3D beam-column elements and rubber bearings as linear 3D spring-dashpots (Figure 2). Based on pier cross-sectional properties, the moments of inertia for fully cracked cross-section have been calculated as equal to 5.805 and 7.253 m^4 for the longitudinal and transverse directions, respectively [9]. The Rayleigh damping is used to model the dissipation of energy in the piers during their vibration. The damping matrix, **C**, of beam-column elements is assumed to be linearly proportional to the stiffness matrix, **K**, according to the formula [10]

$$\mathbf{C} = \alpha \cdot \mathbf{K} \tag{1}$$

where α is a stiffness damping coefficient. Its value has been calculated as $\alpha = 2.1505 \times 10^{-3}$ s [10]. A linear model is used to simulate the behaviour of rubber bearings. The effective stiffness and the equivalent damping ratio for a pair of bearings in the longitudinal and transverse directions are given as $K_b = 2.3298 \times 10^7$ N/m, $\xi_b = 0.14$ and in the vertical direction as $K_v = 1.86657 \times 10^{10}$ N/m, $\xi_v = 0.14$ [8, 11].

Pounding between adjacent superstructure segments is controlled by gap-friction elements placed at the ends of segments (Figure 2). When contact occurs, the ends of segments are fixed in the longitudinal direction of the bridge, and friction forces are imposed in the transverse and

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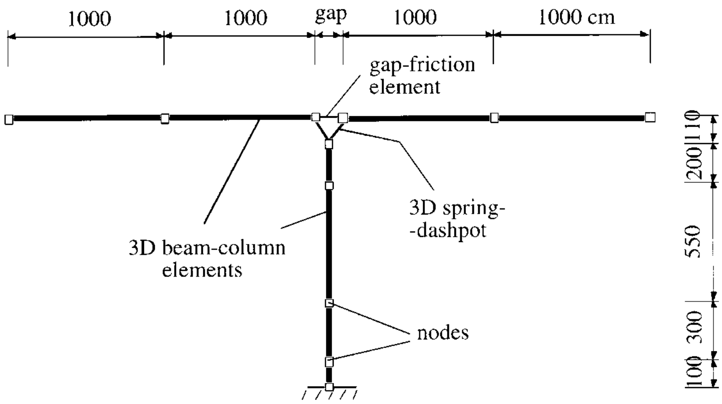

Figure 2. Detailed three-dimensional model of a single pier section with expansion joint.

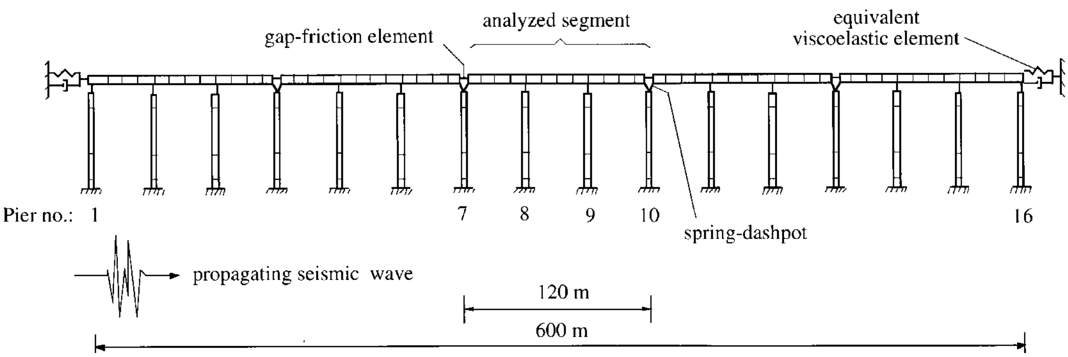

Figure 3. Detailed model of the bridge.

vertical directions. In this study, pounding induced by the seismic wave propagation effect is analysed. This effect is modelled by applying identical input earthquake excitation records acting with time lags on supports along the structure [12, 6].

In this paper, the response of a single superstructure segment of infinitely long bridge is studied. Because of pounding, it depends substantially on displacement histories of few adjacent segments from both sides. By decreasing the number of segments taken into account, it was found [6] that at least five of them should be analysed with neglected parts of the bridge simulated as spring-dashpot elements (with stiffness and damping representing longitudinal properties of a single segment) as shown in Figure 3. For such configuration, the influence of edge conditions is reduced, and the accurate response of the middle segment can be obtained [13].

RESPONSE ANALYSIS

The analysis on the bridge model (Figure 3) is conducted using a step-by-step direct integration Newark-β method with the optimized parameters: $\gamma = 0.5$, $\beta = 0.25$, assuring the stability and

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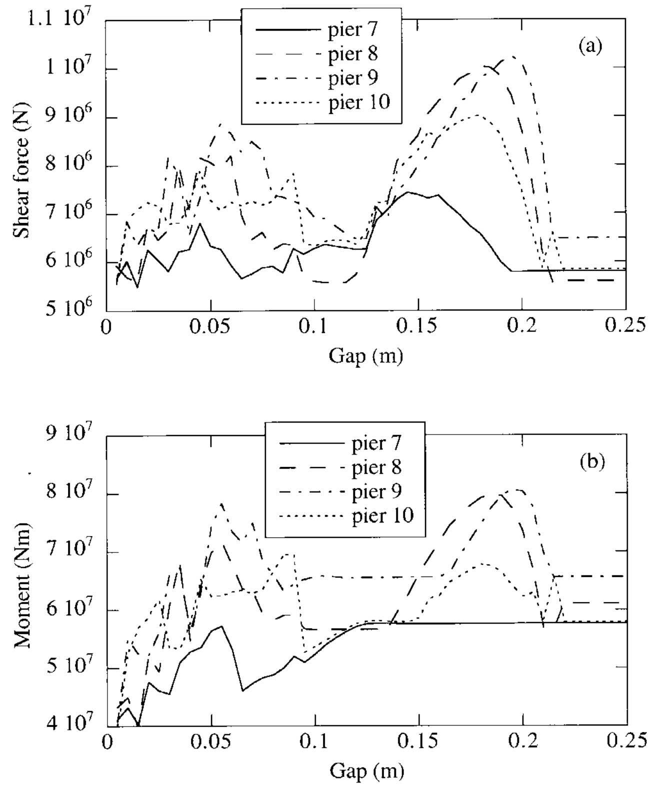

Figure 4. Maximum reaction forces in the longitudinal direction for middle segment piers with respect to gap size between segments: (a) shear forces; (b) bending moments.

accuracy of the results [10]. The time step of 0.005 s is applied to the analysis. The structure is subjected to the longitudinal, transverse and vertical excitations by NS, EW and UD components of the earthquake, respectively. In this paper, the results of the bridge response under the Kobe earthquake (JMA record) of 1995 are presented; however, similar findings can also be obtained for other ground motion excitations [13]. In the analysis, the seismic wave is considered to travel along the bridge with a constant apparent velocity of 1000 m/s.

The maximum shear forces and bending moments at the bases of the middle segment piers with respect to the gap size between segments are shown in Figures 4 and 5 for the longitudinal and transverse directions, respectively. Since yielding of the tensile reinforcement would cause a failure of the pier [9], the minimum vertical axial forces at the piers bases are also presented in Figure 6.

It can be seen from the graphs that the values of the reaction forces can significantly increase compared with the case when pounding does not occur (increase up to nearly 76 per cent for pier No. 8). The results indicate that for very small gaps up to about 0.02 m and large enough to prevent pounding (larger than 0.22 m), the lowest response can be obtained. Additionally, for the Kobe earthquake excitation, smaller values of reaction forces are obtained for the middle gap interval of about 0.09–0.13 m. This is due to the fact that, in this specific gap range, most of the

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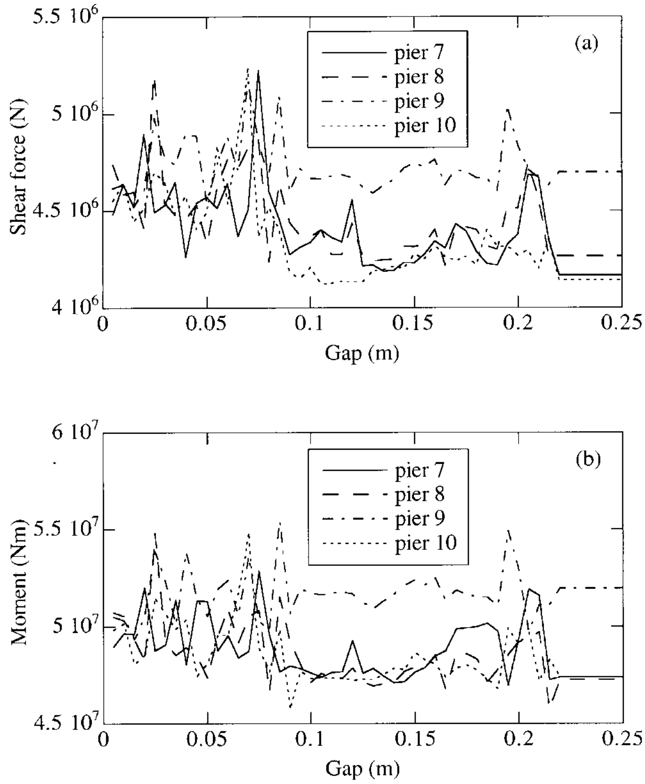

Figure 5. Maximum reaction forces in the transverse direction for middle segment piers with respect to gap size between segments: (a) shear forces; (b) bending moments.

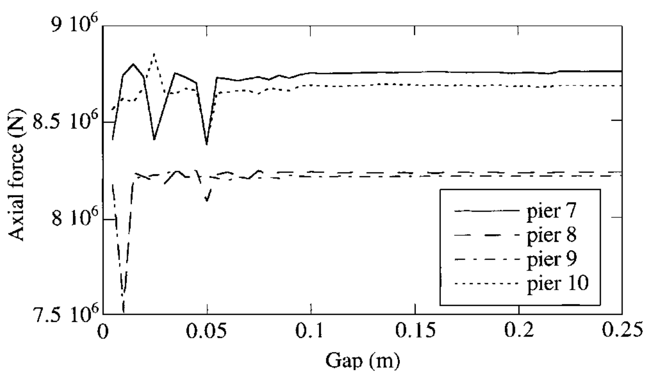

Figure 6. Minimum axial reaction forces of middle segment piers with respect to gap size between segments.

collisions occur when segments are approaching from different directions leading to the reduction of the response. It should be emphasized, however, that the middle gap interval may not be observed for other ground motion excitations [13]. It can be also seen from Figures 4–6 that in

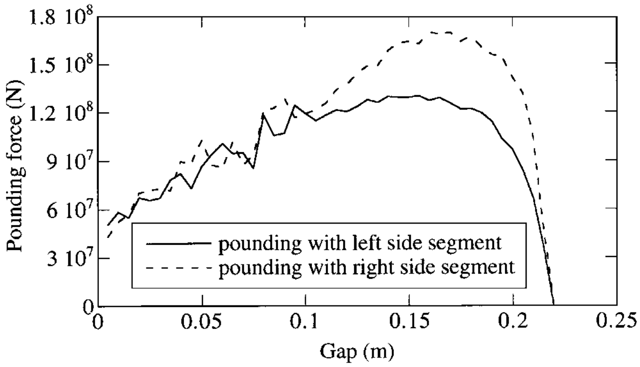

Figure 7. Maximum pounding forces exerted on middle segment with respect to gap size between segments.

the gap size range where pounding is avoided, the maximum reaction forces are different for all analysed piers. This difference in the responses is caused by the seismic wave propagation effect. This effect usually results in smaller reaction forces for piers supporting two different segments since they can move to the opposite directions at the same time and thus the total force exerted on the pier is smaller.

The results from Figures 5 and 6 show that the structural behaviour in the transverse and vertical directions is not significantly influenced by collisions between superstructure segments. Therefore, further analysis presented in this paper is only focused on the longitudinal bridge response.

The maximum pounding forces exerted on the middle superstructure segment due to collisions with the segment from the left and the right side versus the gap size between segments are presented in Figure 7. It can be seen from the figure that with the increase of the gap size up to about 0.17 m, pounding forces uniformly increase. Then, their values fall sharply to zero for gaps large enough to prevent collisions.

OPTIMIZATION OF BRIDGE PROPERTIES

Gap size between superstructure segments

The results of the analysis presented in Figures 4–7 show that for two gap size intervals between adjacent superstructure segments, the smallest structural response can be obtained. They indicate that the optimal gap size is either a very small one or large enough to avoid collisions.

The interval of a very small gap size stands for the case of nearly fully continuous deck. When the seismic wave propagates along the structure, the long continuous deck is subjected in different places to excitations shifted in time. This makes them act in the opposite directions at the same time and thus cancel out their effects on piers. However, expansion joints applied in bridges are designed to accommodate length changes of the superstructure due to thermal and reological (creep, shrinkage) effects. They should also provide appropriate space for placement of deck elements. This usually requires some minimum separation gap, which for the analysed bridge should be as big as 0.05 m [7]. However, it can be seen from Figure 4 that for this value of the gap size, the maximum reaction forces are relatively large.

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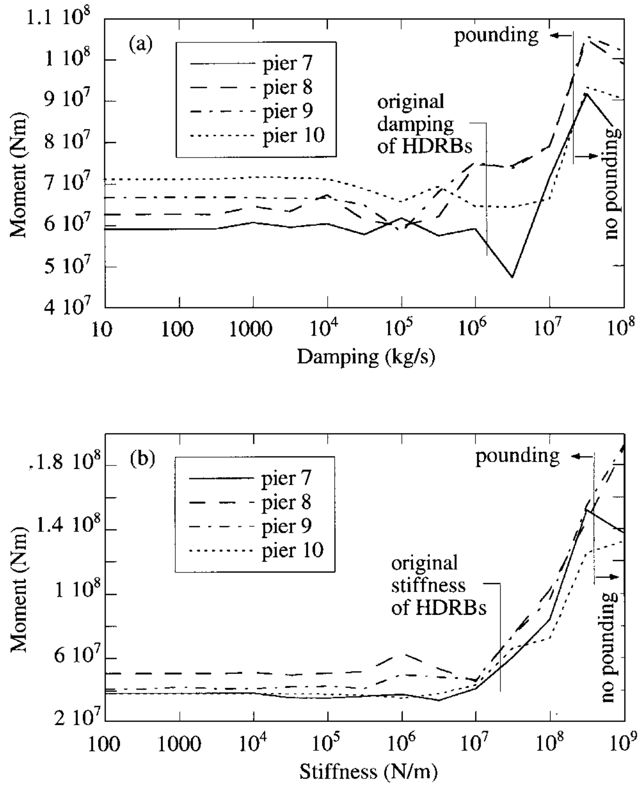

Figure 8. Maximum bending moments of middle segment piers with respect to (a) damping of bearings; (b) stiffness of bearings.

On the other hand, in the case of a large gap size, every superstructure segment vibrates independently and the energy is dissipated through its free movement. Nevertheless, in order to prevent collisions, a significant increase of the separation gap up to about 0.22 m (Figure 7) would be required. However, enlarging the gap between superstructure segments leads to large expansion joint and disturbs traffic on the deck.

Rubber bearings

The bridge performance under different properties of bearings is also analysed. The maximum bending moments of the middle segment piers of the bridge model (Figure 3) with respect to damping and stiffness of a pair of bearings for the example gap size of 0.05 m are shown in Figure 8. It can be seen from the figure that the application of bearings with smaller damping or stiffness properties does not reduce the reaction forces much. Figure 8 also shows that the increase of damping and stiffness of bearings leads to elimination of pounding but at the same time the reaction forces significantly increase (up to nearly 300% for pier No. 7). This is due to the fact that the increase of parameters of bearings makes the structure stiffer and thus less isolated from the ground motion excitation.

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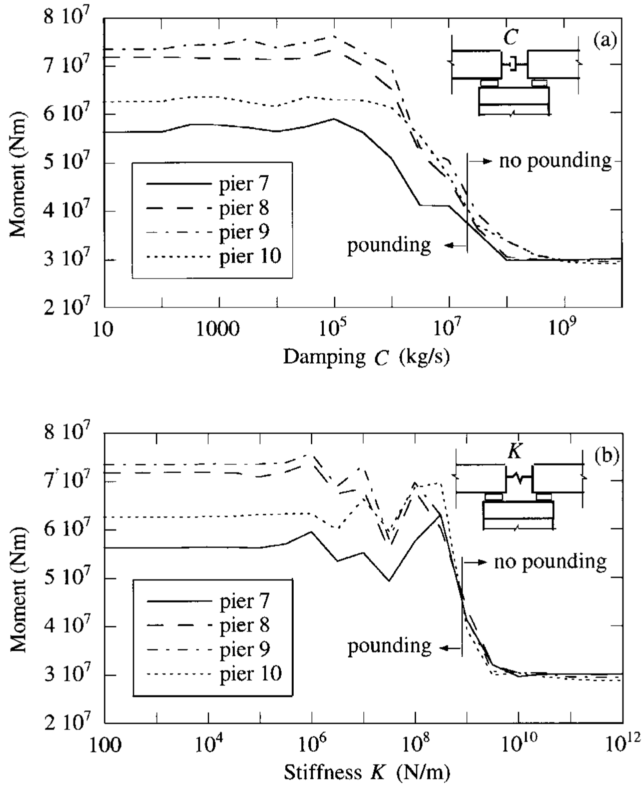

Figure 9. Maximum bending moments of middle segment piers with respect to (a) damping of additional connectors; (b) stiffness of additional connectors.

INCORPORATION OF ADDITIONAL DEVICES BETWEEN SUPERSTRUCTURE SEGMENTS

Since the optimization of the bridge properties does not lead to the improvement of the structural behaviour, the reduction of the negative pounding effects by incorporation of additional devices placed between superstructure segments is investigated. The analysis of the bridge response in the longitudinal direction is conducted for the gap size of 0.05 m between segments.

Dampers and stiffeners

First, the application of additional dampers and stiffeners placed between superstructure segments is considered. The results of the analysis carried out for different parametric values of such connectors are presented in Figure 9. The figure indicates that collisions can be eliminated and the bridge behaviour substantially improved for large damping and stiffness coefficients: $C \geqslant 3 \times 10^7$ kg/s, $K \geqslant 10^9$ N/m. These values, however, represent a nearly continuous deck case. Smaller values do not prevent pounding and moreover may change the pattern of collisions

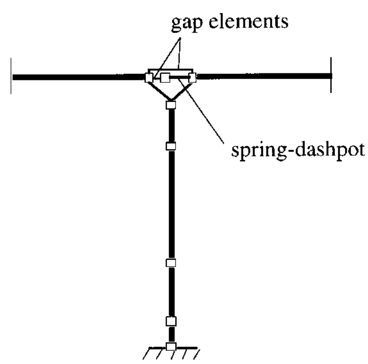

Figure 10. Modification of modelling of expansion joints in bridge model.

between segments. This can even lead to the increase of reaction forces as can be observed in Figure 9(b) for pier No. 7 for the range of stiffness coefficient: $10^6 < K < 10^9$ N/m.

Rubber bumpers

The effective way of reducing the gap between superstructure segments and providing additional damping to the system is to fill the gap with a viscoelastic material such as, for example, rubber. In order to model the behaviour of such rubber bumpers, linear spring-dashpot elements are employed. The stiffness, k_p, of a single device depends on its dimensions and can be calculated from the formula

$$k_p = \frac{E_r A_p}{t_p} \tag{2}$$

where A_p is a cross-sectional area of the pad, t_p its thickness and E_r the Young modulus of rubber. On the other hand, the bumper's damping, c_p, can be computed as [14]

$$c_p = 2\xi_r \sqrt{k_p \frac{m_s}{2}} \tag{3}$$

where ξ_r is a damping ratio of the rubber and m_s is a mass of the superstructure segment.

The analysis of the response of the bridge equipped with rubber bumpers is conducted on the structural model from Figure 3 with modified simulation of expansion joints shown in Figure 10. According to the introduced modification, a linear spring-dashpot element modelling the bumper has been placed between adjacent superstructure segments. To control its deformations, two additional gap elements have been also applied. When the distance between the bumper and the deck end is reduced to zero, the first gap element is activated leading to compression of bumper. If the relative displacement between superstructure segments is further reduced and the ends of segments come into contact, the second gap element becomes active.

The maximum bending moments of the middle segment piers with respect to stiffness of 0.04 m thick rubber bumpers ($E_r = 4.421 \times 10^6$ N/m^2, $\xi_r = 0.14$) placed inside 0.05 m gaps are presented in Figure 11. The pounding force time history between adjacent segments for bumpers with stiffness 10^9 N/m together with the case without bumpers is also shown in Figure 12.

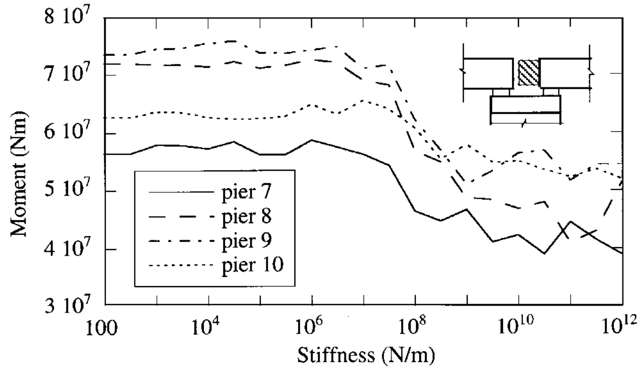

Figure 11. Maximum bending moments of middle segment piers with respect to stiffness of single bumper.

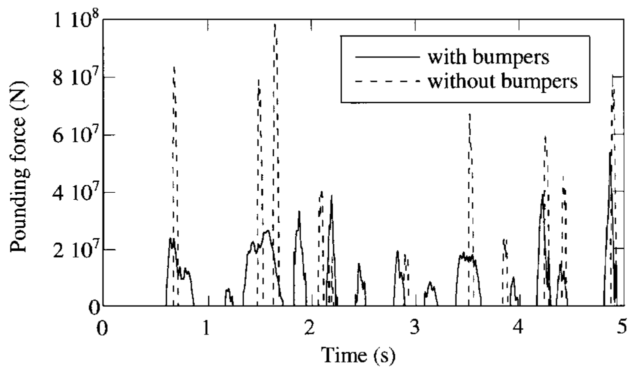

Figure 12. Pounding force time histories with and without bumpers between segments.

It can be observed from Figure 11 that the application of harder bumpers (with stiffness bigger than about 10^8 N/m) leads to the reduction of reaction forces. Moreover, Figure 12 shows that the devices can also significantly reduce the pounding forces exerted on superstructure segments. In this case, due to earlier activation of bumpers, contacts last longer but the maximum forces of collisions are much smaller.

Crushable devices

The methods of reduction of pounding effects by stiff connection of superstructure segments and application of hard rubber bumpers benefit from making the superstructure nearly continuous. As shown earlier, the bridge behaviour can also be improved by allowing the unrestrained vibration of deck segments; however, enlarging the gap size between segments is undesirable. Therefore, in this paper, a concept of applying special crushable devices attached at the end of segments is investigated. Under normal conditions, such devices participate in the transmission of vertical traffic loads. During minor earthquakes, the space to the adjacent superstructure segment is big enough to accommodate the deck movements. In the case of severe ground motion,

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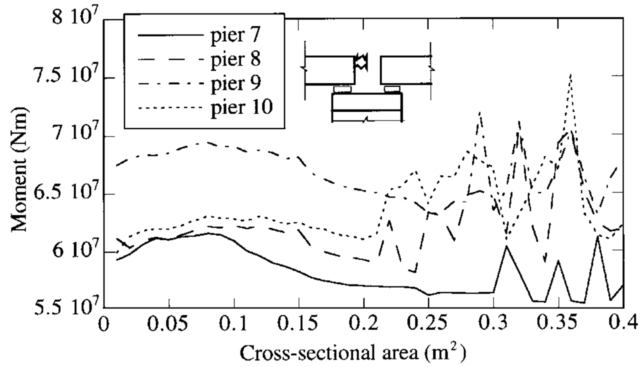

Figure 13. Maximum bending moments of middle segment piers with respect to cross-sectional area of crushable device.

however, the crushable devices are designed to be destroyed by compression, giving sufficient space and absorbing the energy. After the destruction of the devices, the superstructure segments may vibrate independently and smaller structural response can be achieved. Moreover, by the concentration of impact energy in the crushable units, the damage of deck ends, which is difficult and expensive to be repaired, is also prevented. After the earthquake, the destroyed elements can be easily replaced by new items but no repair of structural members is needed.

The study on the behaviour of the bridge incorporating the crushable devices is carried out using a bridge model with similar modification of expansion joints as shown in Figure 10 for the bumper case. In the model, however, instead of a spring-dashpot element, an elastic-perfectly plastic truss member simulating the crushable device, has been applied. Two gap elements have been also used to control the crushing mechanism. When the distance between the ends of the crushable device and the deck is reduced to zero, the first gap element is activated and the device is being compressed. The compression leads to the plastic deformation of the unit if the stress exceeds the yielding value. The second gap element, placed between the ends of segments, is activated when the crushable device has been completely destroyed.

The analysis of the bridge response is conducted for different values of the cross-sectional area of crushable devices. The results for 0.2 m long truss elements made of steel ASTM A441 [15], with yielding point 3.4×10^8 N/m^2 are presented in Figure 13. It can be seen from the figure that, generally speaking, smaller response can be expected for weaker crushable devices (with the cross-sectional area smaller than about 0.21 m^2). The results of the study indicate that the dissipation of energy mechanism due to plastic deformation of crushable devices is not so effective and benefits come from unrestrained vibration of superstructure segments after quick destruction of the devices. It is suggested, therefore, that the crushable devices should be designed to transmit the vertical forces due to traffic loads with possibly small compression resistance.

Comparison of different methods of reduction of pounding effects

The methods of reduction of the negative effects of collisions considered in this paper have been found to be effective in the reduction of reaction forces at the piers bases as well as in pounding

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Table II. Comparison of the effectiveness of different pounding reduction methods.

Method	Parameters used in the analysis	Reduction of max. reaction forces for pier No. 9 (%)	
		Shear forces	Bending moments
Stiff connections of deck segments	Stiffness of connectors: 3×10^9 N/m	44.6	57.9
Rubber bumpers	Stiffness and thickness of bumpers: 10^9 N/m, 0.04 m	26.5	30.7
Crushable devices	Cross-sectional area and thickness of devices: 0.2 m^2, 0.2 m	18.3	11.3

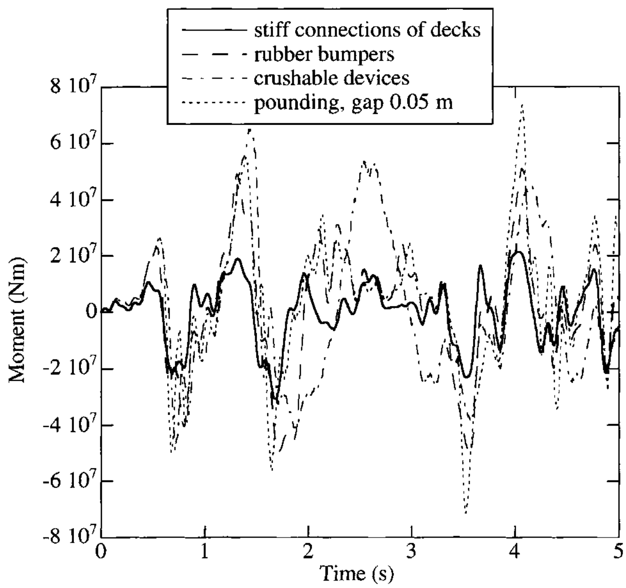

Figure 14. Moment time histories for pier No. 9 for different pounding reduction methods.

forces exerted on colliding superstructure segments. The comparison of the effectiveness of analysed methods for 0.05 m gap size case is shown in Table II. The moment time histories for pier No. 9 of the model of the bridge incorporating different techniques are also presented in Figure 14. It can be seen from Table II and Figure 14 that making the stiff connections of deck segments is the most effective among analysed methods. On the contrary, the worst improvement of the bridge behaviour can be expected when the crushable devices are applied.

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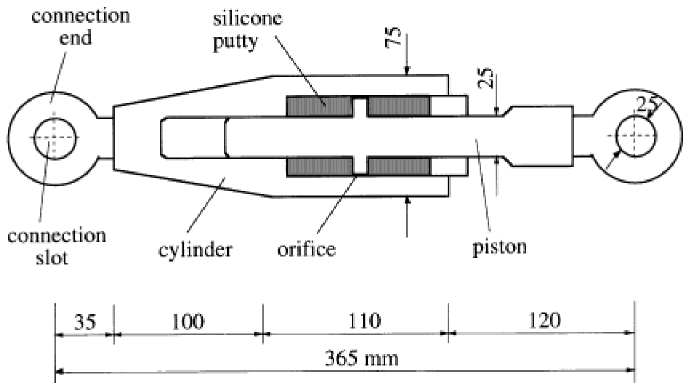

Figure 15. Shock transmission unit (5 tf capacity specimen).

EXPERIMENTAL VERIFICATION OF APPLICABILITY OF SHOCK TRANSMISSION UNITS

Among analysed methods of reduction of the negative pounding effects, the concept of placing additional elements which practically rigidly connect superstructure segments appears to be the most effective one. However, the application of the devices with high damping or stiffness properties makes the superstructure long and continuous. This may result in the increase of internal forces in the deck due to its length changes caused by thermal elongation, creep and shrinkage effects. The desired effect, however, can be obtained by application of shock transmission units (STUs) [16]. The device (Figure 15) is similar to a standard oil damper with the difference of special silicone putty (boron-filled di-methyl syloxane) used instead of oil. The putty has such properties that STU shows nearly no resistance under slow relative movements of segments resulting from thermal and reological changes. On the other hand, the device becomes very stiff if impact force is applied.

The shock transmission units have been used in bridge engineering for several years. Their application concerns mainly retrofitting of existing bridges due to the increase of traction and braking forces of vehicles [16]. This was the reason of strengthening, for example, the single-span Tay bridge in Scotland in which every superstructure segment is supported by a moveable bearing at one side and a fixed bearing at another side. Originally, the vehicle load acting on a given segment had to be transmitted through the fixed bearing to the single pier. After installing STUs between the deck and piers at the moveable bearings locations, the increased vehicle load acting anywhere on the deck is now shared by all the piers simultaneously.

Although, the STUs have been successfully applied in several bridge structures, there are only few experimental results available about the performance of the devices. The basic mechanical behaviour of a 10 tf capacity unit was investigated by Tanzo and Tsuzuki [17]. During the experiment, the specimen was subjected to repeated cycles of force-controlled loading with various amplitudes and frequencies. The experimental results showed the stiffening effect of the unit beyond the certain frequency value which may be controlled by the piston size.

In this study, a device with improved piston-cylinder arrangement of 5 tf capacity (Figure 15) has been tested experimentally. During the experiment, the specimen was subjected to the

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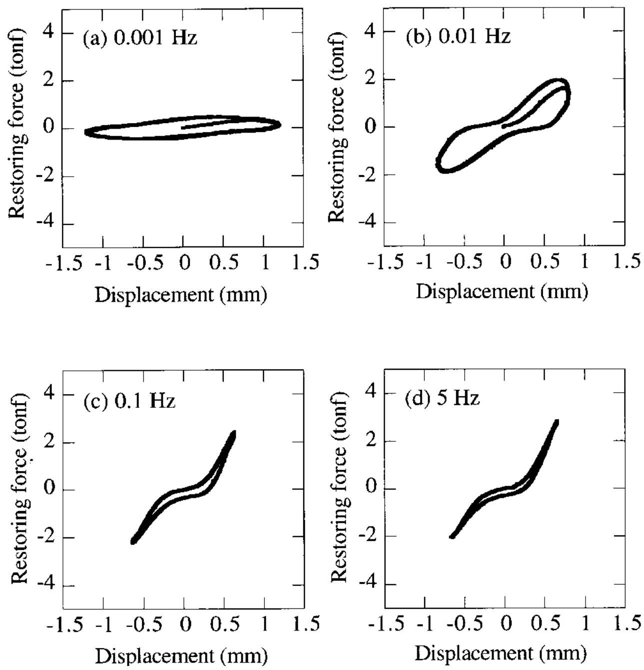

Figure 16. Hysteresis loops of STU for different excitation frequencies.

harmonic excitation of different frequencies. The displacement feedback control with maximum displacement of 1.2 mm was used. The experimental results in means of the load-deformation hysteresis loops for different excitation frequencies are shown in Figure 16.

The figure indicates that when the frequency of 0.001 Hz is applied, the silicone putty can be easily squeezed through the orifice around the piston, generating only small drag forces. When the excitation frequency increases from 0.001 to 0.1 Hz, the stiffening effect of the device can be observed. For the frequency values bigger than 0.1 Hz, no remarkable change in the load-deformation behaviour is recorded. In this case, the putty do not flow through the orifice and the stiffness of STU is provided by properties of the steel cylinder and piston. The slip which can be observed in the transition zone between compression and tension is caused by the clearance take-up of the piston and results from imperfect packing of the putty inside the cylinder.

The results of the experiment suggest that for the modelling purposes of STU under service loads caused by thermal, creep and shrinkage effects, which result in slow movements, the influence of the device can be neglected. On the other hand, the model of STU performance during earthquakes, which cause vibrations with a band of frequencies higher that 0.1 Hz, is proposed in Figure 17. It is assumed that there is no action of the unit within the small range of the piston clearance take-up which is about 0.5 mm. It has been found that exact modelling of observed in this range damping properties of STU does not influence the performance of the bridge equipped with the devices and thus is here neglected. Beyond the piston clearance take-up range, the device is assumed to behave as a linear spring with stiffness provided by steel elements of the unit.

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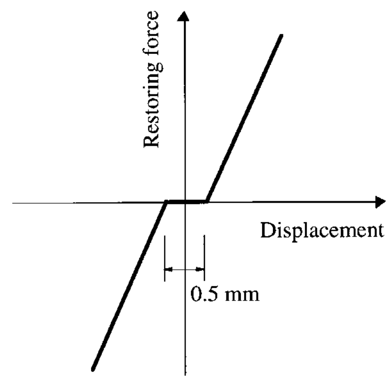

Figure 17. Modelling of STU performance during earthquakes.

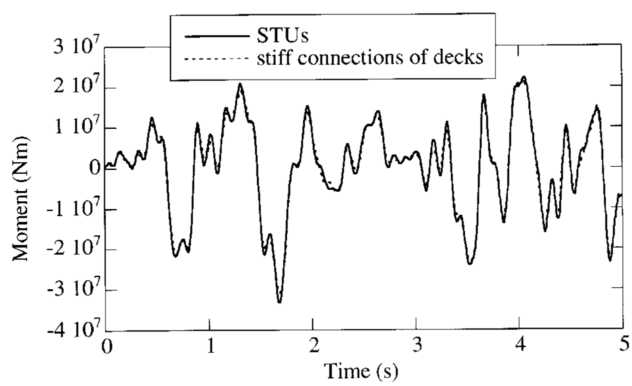

Figure 18. Moment time histories for pier No. 9 in case of STUs placed between segments and stiff connections of decks.

The structural model from Figure 3 incorporating the model of STU performance during earthquakes (Figure 17) is used to study the improvement of behaviour of the bridge with the devices installed between superstructure segments. In order to compare the results with the case of stiff connections of deck segments, the stiffness of STUs beyond the piston clearance take-up range is set at 3×10^9 N/m (see Table II). The moment time histories for pier No. 9 for both cases are shown in Figure 18. It can be seen from the figure that the improvement of the bridge behaviour is slightly worse when STUs are applied which is caused by the slip due to piston clearance take-up. Nevertheless, comparing with the case where pounding is allowed, the application of STUs brings the reduction of 41.4 and 54.6 per cent for maximum shear forces and bending moments at the base of pier No. 9, respectively.

CONCLUSIONS

In this paper, the study of pounding between adjacent superstructure segments of elevated bridges has been conducted, and several methods which can effectively improve the structural behaviour by reducing the negative effects of collisions have been investigated. The analysis has been carried out on a detailed three-dimensional structural component model of an isolated elevated bridge in which collisions are induced by the seismic wave propagation effect.

The results of the study show that the influence of pounding on the structural behaviour is significant in the longitudinal direction of the bridge and depends much on the gap size between superstructure segments. The smallest structural response can be obtained for very small gap sizes and for gap sizes large enough to avoid collisions. However, the application of both intervals is usually an undesirable solution. Further analysis indicates that reaction forces at the piers bases and pounding forces exerted on the superstructure can be satisfactorily reduced by applying simple method of placing hard rubber bumpers between segments. On the other hand, the bridge behaviour can be substantially improved when adjacent segments are fixed one to another. The experimental results indicate that for the practical application of such connectors, STUs can be used. They show minor resistance under slow movements of deck resulting from change of its length and become very stiff during earthquake impact.

In the study presented in this paper, several simplifications concerning the mechanism of collisions and the response of piers have been introduced. Further research including detailed modelling of such effects as damage to the points of contact, non-linear piers behaviour under large displacements and soil–foundations interaction should be conducted. Moreover, this study has been focused on pounding in elevated bridges induced by the seismic wave propagation effect. Further research is therefore needed to extend the knowledge about the influence of collisions in different bridge structures where destructive contacts may occur because of other reasons.

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REFERENCES

1. Priestley MJN, Seible F, Calvi GM. *Seismic Design and Retrofit of Bridges*. Wiley: New York, 1996.
2. Malhotra PK, Huang MJ, Shakal AF. Seismic interaction at separation joints of an instrumented concrete bridge. *Earthquake Engineering and Structural Dynamics* 1995; **24**:1055–1067.
3. Otsuka H, Unjoh S, Terayama T, Hoshikuma J, Kosa K. Damage to highway bridges by the 1995 Hyogoken Nanbu earthquake and the retrofit of highway bridges in Japan. *Proceedings of the 3rd U.S.-Japan Workshop on Seismic Retrofit of Bridges*, Osaka, Japan, 10–11 December 1996.
4. Ma X, Pantelides CP. Nonlinear pounding of bridges in earthquakes. *Structures Congress ASCE*, Chicago, 17–19 April 1996; 1180–1187.
5. Kawashima K, Yabe M. Effectiveness of unseating prevention device with energy dissipation. *Proceedings of the 4th U.S.–Japan Workshop on Earthquake Protective Systems for Bridges*, Osaka, Japan, 10–11 December 1996.
6. Jankowski R, Wilde K, Fujino Y. Pounding of superstructure segments in isolated elevated bridge during earthquakes. *Earthquakes Engineering and Structural Dynamics* 1998; **27**:487–502.
7. Japanese Road Association. *Highway Design Specifications*. Tokyo, 1996 (in Japanese).

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8. Kawashima K, Okado M, Horikawa M. Design example of a highway bridge based on the Manual for Menshin Design of Highway Bridges. *Recent Selected Publications at Earthquake Engineering Division*, *Public Works Research Institute* (*No*. 2), Ministry of Construction, Tsukuba, Japan, May 1993; 191–208.
9. White RN, Gergely P, Sexsmith RG. *Structural Engineering*. *Vol. 3*: *Behavior of Members and Systems*. Wiley: New York, 1974.
10. Chopra AK. *Dynamics of Structures. Theory and Applications to Earthquake Engineering*. Prentice-Hall: Englewood Cliffs, NJ, 1995.
11. Kelly JM. *Earthquake-Resistant Design with Rubber*. Springer: London, UK, 1993.
12. Jeng V, Kasai K. Spectral relative motion of two structures due to seismic travel waves. *Journal of Structural Engineering* 1996; **122**(10):1128–1135.
13. Jankowski R. Pounding of superstructure segments in elevated bridges during earthquakes and reduction of its effects. *Ph.D*. *Dissertation*. Department of Civil Engineering, University of Tokyo, Tokyo, Japan, 1997.
14. Harris CM, *Shock and Vibration Handbook*. McGraw-Hill: New York, 1988.
15. American Institute of Steel Construction. *Manual of Steel Construction*. AISC Inc: New York, 1967.
16. Pritchard BP. The use of shock transmission units in bridging. *Proceedings of the Institution of Civil Engineers*, *Structures & Buildings* 1996; **116**:82–95.
17. Tanzo W, Tsuzuki A. Enhanced seismic lateral load distribution in continuous span bridges fitted with viscoelastic devices. *11th World Conference on Earthquake Engineering*, Acapulco, Mexico, 23–28 June, 1996, paper No. 1528.

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